CHAPTER 6

DEVELOPMENT OF DESIGN CONCEPTS

6-1. Introduction

This chapter describes general procedures for the development of design concepts for the structural upgrading of existing buildings to comply with the acceptance criteria prescribed in chapter 5. Guide lines are provided for the upgrading of the structural systems, the determination of the capacities
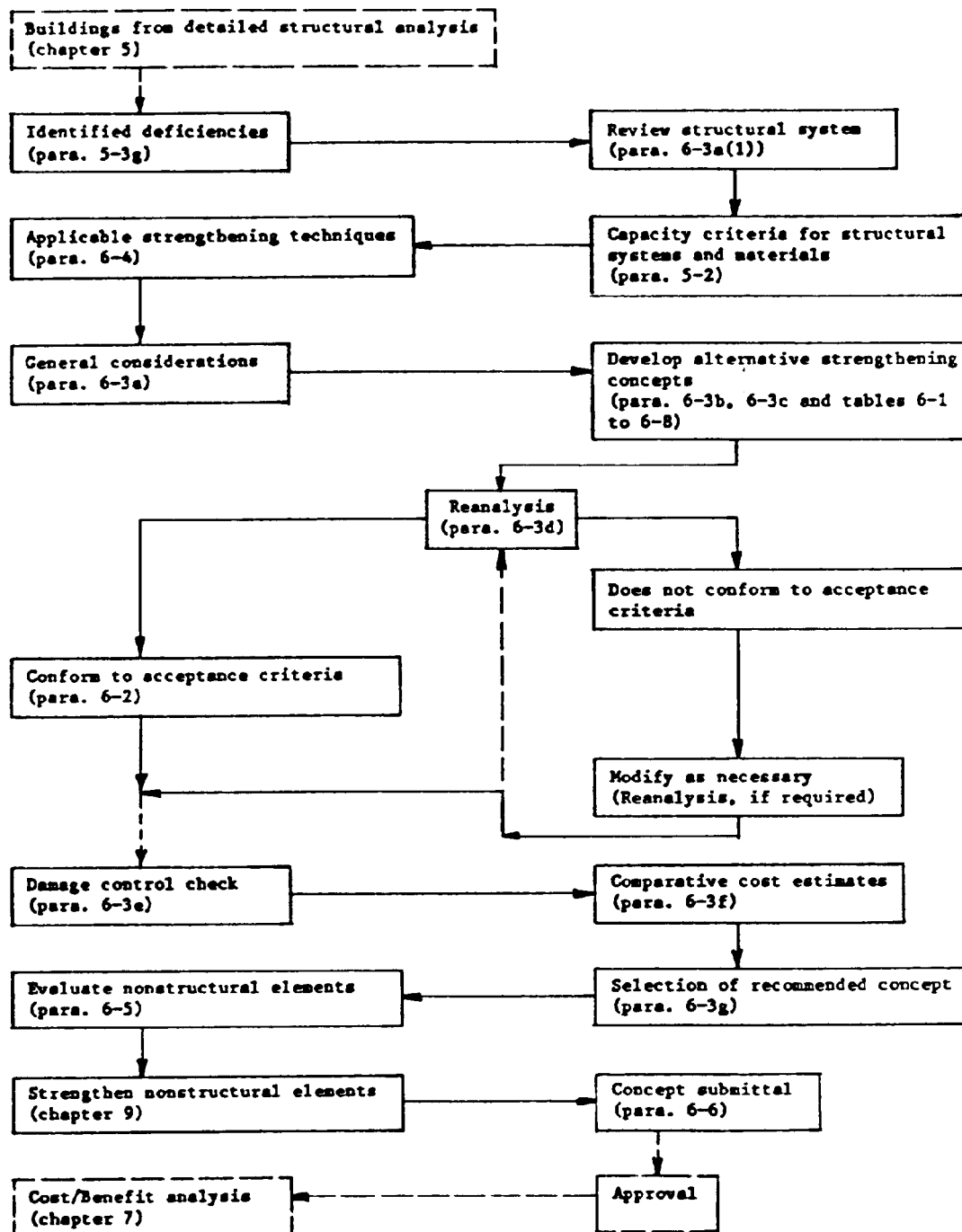

Figure 6-1. Methodology for development of design concepts for seismic upgrading

of new structural elements, and development of strengthening techniques. The methodology for the procedures contained in this chapter is indicated in the flow chart in figure 6-1.

6-2. Acceptance criteria

The design criteria for the development of concepts for seismic upgrading of existing buildings will be in accordance with the applicable provisions of the BDM and/or the SDG as required for new construction. Unless otherwise directed by the approval authority, the minimum acceptance criteria for the conceptual designs will be as indicated in paragraph 5-2 for the detailed structural analysis. The objective of seismic upgrading will be to establish full compliance with SDG provisions for the EQ-II post-yield evaluation (refer to SDG paragraphs 4-4 and 5-5) or, when so directed by the approval authority, the applicable provisions of the BDM or appendix C. In most cases the costs associated with full compliance, as opposed to the reduced force levels permitted by the minimum criteria, will be negligible. However, the allowable reduction will provide a margin for acceptance in those cases where strict adherence to the unreduced criteria would result in much more expensive or disruptive procedures (e.g., a 15 percent reduction in the EQ-II response spectra may make it possible to accept an existing building without strengthening the existing foundations or the construction of an additional shear wall; however, if even with the reduced criteria foundation strengthening or a new wall is required, the upgrading design will be in compliance with the unreduced criteria).

6-3. Development of concepts for seismic upgrading

The results of the detailed structural analysis prescribed in chapter 5 will identify, for a given building, the deficiencies with respect to the acceptance criteria of the various structural elements or systems. These results will be carefully reviewed in the development of alternative design concepts to upgrade the structure to meet the acceptance criteria. Three alternative concepts will be developed for each building unless justification can be shown for fewer alternatives (e.g., it may be shown that the obvious cost effective solutions for a deficient steel braced building are either to replace the existing bracing with stronger bracing members, or to add new bracing so that only two alternatives need to be developed). Each concept will be developed to the extent that will permit a reasonable cost estimate to be made. The extent of removal of existing construction will be indicated; the sizes and locations of new, replaced, or

strengthened structural members will be indicated; typical structural connections will be shown; and the extent and schematic details for upgrading nonstructural elements will be indicated.

a. General considerations. In addition to the acceptance criteria of paragraph 6-2, the following general considerations will be addressed in the development of the design concepts:

- Structural systems
- Plan configuration
- Horizontal diaphragms and foundation ties
- Eccentricity
- Deformation compatibility of new and existing materials
- Inelastic demand ratios
- Drift limits
- Base isolation and energy dissipation

(1) *Structural systems.* The development of the structural upgrading concept requires a complete understanding of the existing vertical and lateral load resisting systems of the existing building. The designer must be able to determine the consequences that the removal, addition, or modification of any structural or nonstructural element will have on the performance of the strengthened building.

(a) Gravity load resisting system. An evaluation of the existing vertical load carrying structural system will be made to determine the effects that the seismic upgrading may have on future performance of the building to resist dead and live gravity loads. The evaluation will include a description of the components of the vertical load carrying system and the load path from the source of the dead and live loads to the foundations.

1. Floor and roof framing. In most buildings, the horizontal framing systems (i.e., floors and roofs) will participate in the lateral force resisting system as a diaphragm in addition to supporting the gravity loads. As part of the seismic upgrading, the floor and roof systems may require modifications (e.g., superimposed diaphragms or horizontal bracing) that will add to the dead load; thus, the capacity of the modified system must be evaluated for the new loading conditions.

2. Vertical structural elements. Vertical load resisting elements such as columns, bearing walls, and framing systems, may also be affected by the seismic upgrading. In addition to the added weight that may be imposed due to the seismic strengthening, these elements may participate in the lateral force resisting system. For example, bearing walls may also be used as shear walls and frames may be strengthened or braced to resist seismic forces. If these framing elements are not used for the lateral force resisting system, they will have to

be analyzed for deformation compatibility. This analysis will include the effects of the lateral displacements due to extreme seismic motion on the vertical load carrying capacity of the vertical structural elements.

3. Foundations. If the seismic upgrading adds weight or redistributes the existing gravity loads, the foundations must be analyzed for the additional gravity loads combined with the horizontal and overturning forces associated with the seismic lateral force design.

(b.) Lateral load resisting systems. The structural system that is designed to resist the seismic forces basically relies on a complete three-dimensional space frame; a coordinated system of shear walls or braced frames with horizontal diaphragms; or a combination of both. Descriptions of these basic systems and their components for new construction can be found in the BDM, paragraph 2-9. In the evaluation and upgrading of an existing structure, it is sometimes difficult to identify an existing lateral force resisting system. Innovative analytical procedures and reliance on existing materials and systems that are not generally considered for new construction are required to determine the load paths and capacities of the existing structures. When an existing structure is not adequate to resist the prescribed lateral forces, as determined by the detailed structural analysis described in chapter 5, strengthening of the existing lateral force resisting system will be required.

(2) *Configuration.* If the existing building is highly irregular in plan configuration or is comprised of units with incompatible seismic response characteristics (e.g., a flexible 6-story steel moment frame connected to a 3-story rigid concrete shear wall unit), severe problems that cannot be resolved by strengthening or upgrading could develop at the connection between two units. In such cases, consideration should be given to separating the two units with a structural expansion joint. Each unit should have a complete system for resisting vertical as well as lateral loads. Structural members bridging the joint with sliding supports on the adjacent unit should be avoided. The criteria for building separations in the SDG (para 4-4*e*(2)*(b)*) apply also to existing buildings. Expansion joints should provide for three-dimensional uncoupled response of each of the separate units of a building, but need not extend through the foundations.

(3) *Horizontal diaphragms and foundation ties.* Every upgraded existing building will have either a rigid or semi-rigid horizontal floor diaphragm as defined in Chapter 5 of the BDM. Roof diaphragms may be flexible or semi-flexible provided they comply with the applicable requirements

specified for those diaphragms in the BDM. Foundation ties between pile caps and caissons will be provided in accordance with paragraphs 3-3(J)3c and 4-8 of the BDM. Existing diaphragms and foundation ties that do not comply with these requirements will be strengthened or replaced in accordance with the guidelines of paragraph 6-4, unless proper justification can be provided for waiving the deficiency.

(4) *Eccentricity.* Provisions shall be made for the increase in shear resulting from the horizontal torsional moment due to an eccentricity between the center of mass and the center of rigidity, as prescribed in BDM paragraph 3-3*(e)*4 and SDG paragraph 4-3*e*(5). In the development of upgrading concepts for existing buildings, when the vertical shear resisting elements must be strengthened, supplemented, or replaced with new elements, consideration will be given to location of new or strengthened elements so as to reduce any eccentricity between the center of rigidity and the center of mass.

(5) *Deformation compatibility of new and existing materials.* The compatibility of the deformation characteristics of the existing elements and the new strengthening elements will be considered in the strengthening design of the building.

(a) Relative rigidities. When lateral forces are applied to a building, they will be resisted by the various elements in proportion to their relative rigidities. The lateral stiffness of a structure is calculated on the basis of the deformation characteristics of the lateral force resisting elements. The structure may be flexible (e.g., a light steel frame) or rigid (e.g., a structure with thick masonry walls). If the structure is to be strengthened to resist seismic forces, the new structural elements must be more rigid than the existing elements if they are to take a major portion of the lateral forces and reduce the amount of force that is taken by the existing elements. Both the relative rigidities and strengths of all lateral force resisting elements must be considered. To illustrate, the following two examples are given:

1. Existing steel moment frame strengthened by diagonal steel bracing. Assume an existing steel moment frame building that has a one-inch story displacement due to seismic forces. Diagonal bracing is added to the moment frames to reduce the lateral displacement to 0.1 inch for the same force level. Thus, it can be estimated that the bracing resists about 90 percent of the lateral force and the frame resists about 10 percent. If the original moment frame can safely resist 10 percent of the seismic forces, the bracing system is

effective. (Note this example neglects the possible increase in the magnitude of the seismic forces due to a decrease in the period of vibration.)

2. Existing brick building strengthened by a steel braced frame system. Assume an existing brick building that has a 0.01-inch story displacement due to seismic forces. A steel bracing system is added that has a 0.02-inch story displacement for the same force level. In this case, it appears that after strengthening the building, the brick walls will resist approximately two-thirds of the lateral forces until they fail and transfer load to the steel braced frames. Therefore, the steel bracing system is fully utilized prior to cracking of the brick walls; however, if it subsequently can resist the total seismic forces, it will limit the lateral displacements and prevent excess damage to the brick walls (see subpara (7) below for drift limitations).

(b) Deformation characteristics of structural elements. The accuracy of the relative rigidity calculations is dependent on the accuracy of the assumptions used for determining the stiffness characteristics of each element or system. When all of the lateral force resisting elements are of the same material and have similar deformation characteristics (e.g., flexural and/or shearing deformations), the relative rigidities can be calculated with reasonably good accuracy. However, when there is a variety of materials and cross-sectional shapes, the confidence level on the accuracy of the relative rigidities is greatly reduced. When the confidence level is low, the range of stiffness values should be enveloped and the distribution should be overlapped to account for the inaccuracies. Mathematical modeling guidelines are given in SDG paragraphs 5-4*b* and 5-5*a*(2). Structural elements that require special consideration in determination of relative rigidities include:

Concrete: cracked vs. uncracked

Shear walls: participation of intersecting walls (e.g., effective flange widths) and effects of openings.

Steel frames: participation of concrete fireproofing, floor slab and framing, and infill walls (structural and nonstructural).

(c) Evaluation of structural elements not part of the lateral force resisting system. Structural elements that are not part of the lateral force resisting system will be evaluated for the effects of the deformation that occur in the lateral force resisting system. These provisions for the EQ-II deformations parallel the deformation compatibility provisions in BDM paragraph 3-3(J)1d.

(d) Protection of existing brittle elements. Brittle elements that are not part of the lateral force

resisting system are susceptible to damage if they are forced to conform to the deformations of the lateral force resisting system. In order to protect these elements from the possibility of being subjected to large distortions, provisions can be made to allow the structural system to distort without forcing distortion on the brittle elements. An example of isolating a masonry wall from the slab soffit is shown in the BDM, figure 9-1. When rigid walls are locked in between columns, a similar method of isolation may be required at each end of the wall.

(6) Inelastic demand ratios. Seismic capacity, demand, and inelastic demand ratios shall be calculated in accordance with the provisions of chapter 4 of the SDG and shall not exceed the values given in table 5-1 unless they are supported by other systems that can resist the required lateral forces. For example, in an existing unreinforced masonry bearing wall building with new reinforced concrete shear walls or steel bracing, the masonry walls are assumed to share the lateral forces in proportion to their relative rigidities until the allowable inelastic demand ratio for reinforced masonry is exceeded. At that point, the entire story shear must be resisted by the new shear walls or steel bracing. The masonry wall may be assumed to be capable of supporting the imposed vertical loads, providing the drift limits specified in the following subparagraph are not exceeded.

(7) Drift limits. Lateral deflections, or drift, of a story relative to its adjacent stories for EQ-II will be in accordance with the provisions of paragraph 5-2*a*, except that for unreinforced concrete or masonry walls and nonductile reinforced concrete frames where the allowable inelastic demand ratios are exceeded (see subpara (6) above), the interstory drift limits for the EQ-II forces will be reduced to those given in paragraph 5-2*b* (i.e., 60 percent of para 5-2*a*) unless the above nonductile elements are properly anchored to a new structural system (i.e., reinforced concrete or masonry wall, braced steel frame, etc.) that is capable of resisting the entire story shear.

(8) Base isolation and energy dissipation.

(a) Base isolation. Design strategies that significantly modify the dynamic response of a structure at or near the ground level, are generically termed base isolation. This is usually achieved by introduction of additional flexibility at the base of the structure. The objective is to force the entire superstructure to respond to vibratory ground motion as a rigid body with a new fundamental mode based on the stiffness of the isolation devices. This strategy is particularly effective for short stiff buildings (i.e., with a fundamental mode less than

1 sec). For these buildings, it is feasible with the isolation devices to develop a new fundamental mode with a period of about 2 sec. For most sites (e.g., those with a predominant site period less than 1 sec), the new fundamental mode period will be beyond the portion of the response spectrum that is subject to dynamic amplification and the response of the structures will be greatly reduced. The concept of base isolation is not new; for many years it has been used to reduce the vibration of equipment and machinery with springs, resilient mountings, and shock absorbers. Similarly, bridges and other simple structures have been isolated to reduce vibration and noise, and in some instances, to reduce the seismic excitation. The application to complex structures, such as buildings, has been made possible in recent years due to greatly improved computational capability (e.g., high speed, large capacity, digital computers) and development and marketing of the isolation assemblies. A typical installation consists in large pads of natural or synthetic rubber layers bounded to steel plates in a sandwich assembly. The isolator assembly, as well as all connecting elements and building services, must be capable of resisting the design spectral displacement corresponding to the new fundamental mode (a recent California installation has base isolation assemblies that can deflect elastically up to 18 inches). Some base isolation assemblies may have a lead core or other devices to increase damping and thus decrease the response at the isolator. Because of the uncertainties associated with ground motion predictions, most seismic base isolators are designed with fail-safe provisions to arrest the motion of the building prior to development of instability due to excessive displacement of the isolator. Base isolation can be an effective strategy to reduce the seismic response of buildings provided careful consideration is given to the amplitude and frequency content of the expected ground motion; the design of the connecting services to accommodate the expected displacements; and provision of fail-safe mechanisms as described above. The ability of base isolation to reduce seismic response is even more attractive in application to existing buildings with inadequate seismic resistance. However, in addition to the considerations described above, installation of base isolation in an existing building entails accurate determination of the magnitude and location of the vertical loads; a rigid diaphragm above the isolators to collect and distribute the lateral loads; and careful underpinning and jacking of the existing structure in order to effect a systemic transfer of the existing structure in order to effect a systematic

transfer of the existing foundation loads to the base isolation device. Base isolation has been investigated for a number of existing structures (base isolation for an historic structure in Salt Lake City is currently under construction), and there are provisions to establish construction feasibility or cost-effectiveness of base isolation for seismic upgrading.

(b) Energy dissipation. An effective means of providing a substantial level of damping is through hysteretic energy dissipation. Some structures (e.g., properly designed ductile steel and concrete frames) exhibit additional damping and reduced dynamic response as a result of the limited yielding of structural steel or concrete reinforcement. Mechanical devices, designed to increase structural damping, have been developed using mild steel in flexure or torsion and the deformation of lead by extrusion or shear. Viscoelastic materials in shear have been used successfully to control wind vibration in tall buildings and similar installation have been proposed for reducing the seismic response of buildings. Friction is another source of energy dissipation that can be used to reduce dynamic response and limit deflections. However, frictional resistance is difficult to quantify and its reliability may be questionable. Hydraulic damping has been successfully used on machinery and bridge structures, but there are no known applications used to modify building response. The use of structural dampers to reduce the seismic response of existing buildings may be feasible and cost-effective. The installation of viscoelastic structural dampers as an alternative upgrading concept for design example F-3 has been developed in a recent technical article (see biblio Scholl, R.E.).

b. Selection of strengthening technique.

(1) *General.* The selection of an appropriate strengthening technique for the upgrading of an existing building that does not comply with the acceptance criteria of chapter 5 will depend upon the type of structural systems in the existing building, the nature of the deficiency, and the considerations given in subparagraph *a* above. In some cases, the selection may be influenced by other than structural considerations. For example, a requirement that the building be kept operational, with minimal interference to the functions that it provides during the structural modifications, may dictate that the modifications be restricted to the periphery of the building with as much work as possible accomplished on the exterior of the buildings. On the other hand, it may be possible temporarily to relocate the function and occupants of an existing building that is to be upgraded. This, of course, provides more latitude in the selection of

appropriate and cost effective strengthening techniques. In many cases, seismic upgrading is accomplished concurrently with functional alterations, renovation, and/or energy retrofits. In these cases, the selected structural modification scheme should be the one that best suits the requirements of all the proposed alterations.

(2) *Examples.*

(a) An existing unreinforced masonry building with inadequate shear capacity in walls has reinforced concrete floor and roof diaphragms that are adequate in shear capacity, but do not have adequate chord strength for the flexural action of the diaphragms. A rigid system of new reinforced concrete shear walls or steel braced frames will be required to provide the additional strength and rigidity to protect the masonry walls. Because a chord has to be developed for the existing diaphragms, it may be advantageous to consider strengthening the masonry wall with a new reinforced concrete wall on the inside of the masonry walls as described in paragraph 6-4*b*(4). This will facilitate anchorage of the masonry walls for out-of-plane forces, development of a new diaphragm chord, and shear transfer from the existing diaphragms. A portion of the masonry wall may be removed to reduce the loads on the existing foundations. If the full thickness masonry walls can span vertically for the seismic out-of-plane forces, consideration can be given to providing the new concrete walls in selected locations while minimizing the eccentricity between the center of mass and the new center of rigidity. The diaphragm chords must be continuous to resist the horizontal flexural stresses in the diaphragms and to provide the necessary support to the masonry walls.

(b) A two-story steel frame building has 7 frames in the transverse direction and 3 frames in the longitudinal direction. Three of the 7 frames in the transverse direction and 2 of 3 frames in the longitudinal direction are moment frames. The floor and roof diaphragms consist of steel decking without concrete fill. The existing frames and diaphragms are adequate for the acceptance criteria in the longitudinal direction. The 3 existing transverse frames do not meet the acceptance criteria for drift and the diaphragms do not have adequate shear capacity in the transverse direction for the three bays between the moment frames, but they would be adequate for only two bays. There are a wide range of possible solutions to this example: the diaphragms could be strengthened by adding a concrete fill (para 6-4*b*(7)*(c);* the existing transverse moment frames could be strengthened (paras 6-4*b*(1)*(a)* and *(b);* some of the intermediate transverse frames could be made moment-resisting (para 6-4*b*(1)*(d)*); or new reinforced concrete shear walls or steel bracing could be added to reduce the

drift. It should be noted that modifying the intermediate frames for moment resistance may not be feasible because of interferences with the steel decking. Adding concrete fill to strengthen the steel decking will require removal and replacement of the second floor and the roof. New concrete shear walls will require new foundations. Considerable cost saving can be achieved by eliminating or minimizing the work on the roof, floor, and foundations. Vertical steel bracing becomes a logical solution. If the bracing is installed at the end transverse frames and at every other frame in between, the existing diaphragm will only have to span two bays and will not need to be strengthened. The bracing will be effective in reducing drift, but the resulting shorter period will probably increase the seismic demand forces that now will be resisted primarily by bracing. However, with 4 lines of bracing, the forces are well distributed and the additional foundation loads (shear and overturning forces) may not be difficult to accommodate with the existing foundations.

(c) An existing two-story office building that performs an essential function must be seismically upgraded. The building has an identified deficiency in the transverse direction in which the lateral forces are resisted by nonductile concrete frames. The detailed structural analysis indicates that additional lateral load resistance is required and also that building deformation must be limited so that allowable drift and inelastic demand ratios for the nonductile frames are not exceeded. The structural modification must be accomplished with minimal interference to the functions and occupants in the building. The above restrictions dictate that the optimum solution would be one that provides significant rigidity and can be implemented from the exterior of the building. If the floor and roof diaphragms comply with the requirements of chapter 5 of the BDM, an appropriate scheme for the upgrading would be to provide bracing or shear walls at each end of the building. A potential problem with this scheme might be inadequate resistance to the overturning forces at the foundation level. Possible solutions to the overturning problem would be larger footings; drilled piers to provide tension tie-down; buttresses in the plane of the end walls to increase the resisting lever arm for the overturning moments; or internal shear walls to reduce the lateral forces on the end walls. Prefabricated steel shear walls (para 6-4*d*(2)*(b)*) can be used to minimize the time and area of disruption in an existing building.

(d) An existing three-story unreinforced masonry building is to be seismically upgraded because of the historical significance of its external architecture. The building will be used as an administration building after the seismic upgrading.

The roof and floor systems are timber post and beam construction and the timber flooring is inadequate for diaphragm action or to anchor the walls. Retention of the timber framing will require installation of a sprinkler system. In this building consideration should be given to reconstruction of the second floor and roof systems in reinforced concrete. The existing exterior walls may need temporary shoring while a new reinforced concrete wall is constructed at the interior face of the existing walls (para 6-4*b*(4)). The new walls and floor and roof framing will provide the lateral force systems and will provide the necessary support to the existing exterior walls.

c. Strengthening options for structural systems. Paragraph 6-4 describes various generic strengthening techniques for structural elements. The preceding subparagraph provides two representative examples for the selection of appropriate strengthening techniques that are compatible with the existing structural system and will correct the deficiencies identified in the detailed structural analysis. Tables 6-1 to 6-8 provide a listing of various strengthening options that may be considered for seismic upgrading. This listing should not be considered to be complete or exclusive and the engineer is encouraged to use his initiative in the development of the three required alternative upgrading concepts from the options described in the tables or by innovative variation of those options.

d. Reanalysis. Each alternative upgrading concept will be evaluated for compliance with the acceptance criteria in chapter 5. Unless the effects of the structural modifications on the mass, stiffness, and load distribution in the building are obviously negligible, a reanalysis of each concept will be required. The reanalysis will be similar to the detailed structural analysis but with the revised structural model resulting from the upgrading modifications. In most cases the effect of strengthening and/or stiffening of an existing building will reduce the modal periods of vibration and increase the spectral demand on the building. One or more analysis iterations may be required to reconcile the modified capacity of the building with the seismic demand.

e. Damage control check. After each alternative upgrading concept has been checked for compliance with the acceptance criteria for EQ-II forces, it will also be checked as follows for essentially elastic response to EQ-I forces:

(1) *EQ-I analysis.* Perform an EQ-I analysis in accordance with paragraph 4-3 of the SDG. The acceptance criteria prescribed in paragraph SDG 4-3*e*(1) will be modified as follows:

(a) *Ductile framing systems.* The 25 percent tolerance allowed in excess of the flexural elastic capacity for a limited number of elements may be increased to 30 percent.

(b) *Other framing systems.* The 10 percent tolerance allowed in excess of the flexural elastic capacity may be increased to 15 percent.

(c) *Box systems.* These systems may not exceed the elastic capacity requirements of the SDG.

(2) *Alternatives to EQ-I analysis.* When the EQ-II reanalysis prescribed in paragraph d above has been performed by Method 2, compliance with EQ-I requirements may be made by comparing the elastic capacities, calculated for the EQ-II reanalysis, with the EQ-I spectral requirements. When the EQ-II reanalysis has performed by Method 1 (conventional elastic analysis), the following procedures may be used:

(a) Compare the response spectrum for the EQ-I elastic response to that for EQ-II post-yield response. Determine the spectral acceleration ordinate, S_{aI}, at the building's fundamental period on the EQ-I spectrum and the corresponding ordinate, S_{aII}, on the EQ-II spectrum.

(b) Calculate the ratio, $R = \dfrac{S_{aI}}{S_{aII}}$

(c) Examine a representative sample of inelastic demand ratios (IDR) at each level of the building.

(d) Determine what portion of each IDR is attributed to seismic response as opposed to response to the vertical gravity loads (e.g., for shear walls with an IDR of 1.50, the entire amount may be due to seismic loads where a concrete or steel frame column with the same IDR may have 0.60 due to gravity loads and 0.90 due to seismic loads).

(e) Multiply the seismic portion of the IDR by the ratio, R, previously calculated and add this to the gravity load portion of the IDR (e.g., for a given building, if R = 0.40, the same shear wall with an IDR of 1.50 for EQ-II would have an IDR of 0.60 (i.e., 0.40 x 1.50) for EQ-I while the above frame column would have an IDR of 0.96 (i.e., 0.60 + 0.40 X 0.90)).

(f) Unless adjustments are made for differences in EQ-I and EQ-II gravity load factors, none of the tolerances permitted for exceeding the flexural elastic capacity in paragraph 4-3*e*(1) of the SDG and paragraph (1) above will apply.

f. Comparative cost estimates. After it has been confirmed that each alternative concept is in compliance with the acceptance criteria, comparative cost estimates will be prepared to provide a basis for the selection of the recommended concept. Since the primary purpose of these estimates is to differentiate the relative costs of the concepts, a complete cost estimate is not required at this point. Only those principal items of cost that vary among the concepts need to be recognized. For example,

Table 6-1. Strengthening options for unreinforced concrete or masonry buildings. (Sheet 1 of 2)

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
a. Shear walls	(1) Inadequate shear capacity	(a) Add reinforced concrete to exterior face	Para. 6-4b(4)	Figure 6-7
		(b) Add reinforced concrete to interior face	Para. 6-4b(4)	Figure 6-10
		(c) Remove and replace with reinforced concrete	Para. 6-4c	Figure 6-22
b. Connecting beams	(1) Inadequate shear or flexural capacity	(a) Fill-in openings	Para. 6-4b(3)(a)	Figure 6-9
		(b) Remove and replace with reinforced concrete	Para. 6-4b(3)(a)	Figure 6-8
c. Concrete floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with new reinforced concrete slab	Para. 6-4b(7)(b)	Figure 6-16
	(2) Inadequate chord capacity	(a) Add new reinforced concrete chord	Para. 6-4b(7)(b)	Figs. 6-10, 6-16, 6-17
	(3) Inadequate wall anchorage	(a) Provide drilled-in wall anchors to new construction	Para. 6-4b(4)	Figs. 6-7, 6-8, 6-10, 6-16, 6-17
d. Timber floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with plywood	Para. 6-4b(7)(a)	Figure 6-14
	(2) Inadequate chord capacity	(a) Add new continuous steel angle	Para. 6-4b(7)(a)	Figure 6-14
	(3) Inadequate wall anchorage	(a) Provide drilled-in wall anchors to new construction	Para. 6-4b(7)(a)	Figure 6-14

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Table 6-1. Strengthening options for unreinforced concrete or masonry buildings. (Sheet 2 of 2)

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
e. Spread footings	(1) Inadequate load capacity	(a) Underpin existing footings	Para. 6-4b(8)(a)	Figure 6-20
		(b) Remove and replace with new footings	Para. 6-4c	Figure 6-22
f. Pile or drilled pier footings	(1) Inadequate load capacity	(a) Add additional piles or piers. Remove and replace existing caps.	Para. 6-4b(8)(b)	Figure 6-21

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if diagonal bracing and eccentric bracing are considered as alternate concepts to be installed in the same number of bays and both concepts required the same strengthening of floor and roof diaphragms and foundations, only the differential costs of the two bracing systems need to be identified.

g. Selection of recommended concept. The optimum concept will be the one that meets the acceptance criteria and best satisfies the general considerations of subparagraph a at a reasonable cost. This may not necessarily be the least expensive concept if justification can be provided for greater reliability, improved structural performance, functional advantages, or reduced maintenance of a better and more cost effective system.

6-4. Strengthening techniques

Techniques for strengthening or upgrading existing buildings will vary according to the nature and extent of the deficiency, the configuration of the structural system, and the structural materials of which it is comprised. It is not practicable within the scope of this manual to deal with every possible variation of all conditions. This paragraph will provide guidelines for the seismic upgrading of typical structural members or systems and guidance for structural engineers to utilize judgment and ingenuity to deal with specific situations. The strengthening or seismic upgrading of the building will generally fall into one or more of the following categories: rehabilitation of existing structural members; modification of existing structural members; replacement of existing deficient structural members; or the addition of new structural members or elements (i.e., shear walls, braced frames, etc.).

a. Rehabilitation of existing structural members. Seismic upgrading of existing buildings by strengthening or replacement of existing structural members and/or the addition of new structural members may also require rehabilitation to restore the initial capacity of existing structural members that have been subjected to damage or deterioration. Representative examples of feasible rehabilitation for typical structural members are described in appendix E. General deterioration of materials, such as corrosion of structural steel members or concrete reinforcement, or weathering of concrete brick or mortar, may not be readily repaired and such materials will be assigned a capacity reduction factor as indicated in appendix E.

b. Modification/strengthening of existing structural members. In some cases, the modification and/or strengthening of existing structural members could be the most cost effective method for the seismic

Table 6-2. Strengthening options for reinforced concrete or masonry shear wall buildings. (Sheet 1 of 2)

Structural Element	Deficiency		Strengthening Technique	Reference	Applicable Figure
a. Shear walls	(1) Inadequate shear capacity	(a)	Add reinforced concrete to exterior face	Para. 6-4b(3)(a)	Figure 6-7
		(b)	Add reinforced concrete to interior face	Para. 6-4b(3)(a)	Figure 6-10
		(c)	Fill-in openings	Para. 6-4b(3)(a)	Figure 6-9
		(d)	Add new interior concrete shear walls	Para. 6-4d(2)(b)	Figure 6-24
		(e)	Add new interior steel shear walls	Para. 6-4d(2)(b)	Figure 6-25
		(f)	Add new exterior steel or concrete buttresses	Para. 6-4d(5)	Figure 6-28
		(g)	Remove and replace with new construction	Paras. 6-4b(3)(a) & 6-4c	Figure 6-22
b. Connecting beams	(1) Inadequate shear or flexural capacity	(a)	Fill-in openings in shear walls	Para. 6-4b(3)(a)	Figure 6-9
		(b)	Remove and replace with reinforced concrete	Para. 6-4b(3)(a)	Figure 6-9
c. Concrete floor roof diaphragms	(1) Inadequate shear capacity	(a)	Overlay with new reinforced concrete slab	Para. 6-4b(7)(b)	Figure 6-16
	(2) Inadequate chord capacity	(a)	Add new reinforced concrete chord	Para. 6-4b(7)(b)	Figs. 6-10, 6-16, 6-17
	(3) Inadequate wall anchorage	(a)	Provided drilled-in wall anchors	Para. 6-4b(4)	Figs. 6-7, 6-8, 6-10, 6-16, 6-17

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Table 6-2. Strengthening options for reinforced concrete or masonry shear wall buildings. (Sheet 2 of 2)

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
d. Timber floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with plywood	Para. 6-4b(7)(a)	Figure 6-14
	(2) Inadequate chord capacity	(a) Add new continuous steel angle	Para. 6-4b(7)(a)	Figure 6-14
	(3) Inadequate wall anchorage	(a) Provide drilled-in wall anchors	Para. 6-4b(7)(a)	Figure 6-14
e. Spread footings	(1) Inadequate load capacity	(a) Underpin existing footings	Para. 6-4b(8)(a)	Figure 6-20
		(b) Remove and replace with new footings	Para. 6-4c	Figure 6-22
f. Pile or drilled pier footings	(1) Inadequate load capacity	(a) Add additional piles or piers. Remove and replace existing caps.	Para. 6-4b(8)(b)	Figure 6-21

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upgrading of an existing building. Typical examples of the structural modification of existing structural members, in place, are provided in the following paragraphs. *Other cost-effective methods will be investigated for each condition illustrated in figures 6-2 through 6-28.*

(1) *Structural steel framing.*

(a) Columns. The capacity of columns is determined from interaction equations for axial loads and bending, thus the seismic capacity of a column can be upgraded, within reasonable limits, by increasing either or both its capacity for axial loads or for moment. The axial load capacity of steel columns can be upgraded by welding cover plates on the flanges or by "boxing" the column with plates between the tips of the flanges. Typical details are indicated on figure 6-2. These plates may also serve to increase the moment capacity of the columns at the base. Increasing the moment capacity of existing columns at the beam-column connection is usually not feasible because of the interference of the connecting beams. In some cases, it may be possible to increase the shear capacity of the column web with doubler plates as indicated in figure 6-2 provided that there is adequate clearance for the necessary welding.

(b) Beams. Strengthening of existing beams, in place, may be required to improve the moment capacity by an increase in the section modulus, 5, or to reduce drift by an increase in the moment of inertia, I. The section modulus of a beam may be increased by welding cover plates to the top or bottom flanges. In many cases, it may not be feasible to provide cover plates on the top flange because of interference with the floor beams, slabs, or metal decking. (Note that for a bare steel beam, a cover plate on only the lower flange may not significantly reduce the stress in the upper flange.) However, if the floor slab or metal decking is adequately detailed for composite action at the end of the beam, it may be economically feasible to increase the moment capacity by providing cover plates at the lower flanges at each end of the beam as indicated in figure 6-3. The length of the cover plates, in this case, will be determined from the combined (DL + LL + EQ) demand moment diagram. The cover plates will be tapered as shown to avoid an abrupt change in section modulus beyond the point where the additional section modulus is required. Where frame drift governs, it may be feasible to increase the moment of inertia and thus reduce the drift by the addition of a cover plate to the lower flange of existing steel beams between the columns as also indicated in figure 6-3. It should be noted that beams with discontinuous cover plates must be treated as tapered or haunched sections and will have different

Table 6–3. Strengthening options for reinforced concrete frame buildings.

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
a. Frames	(1) Inadequate lateral load capacity	(a) Add new external steel frames	Para. 6-4d(1)(a)	Figure 6-23
		(b) Add new interior concrete shear walls	Para. 6-4d(2)(b)	Figure 6-24
		(c) Add new interior steel shear walls	Para. 6-4d(2)(b)	Figure 6-25
		(d) Add new exterior concrete or steel buttresses	Para. 6-4d(5)	Figure 6-28
		(e) Structural addition(s) to building	Para. 6-4d(6)	
		(f) Remove and replace with new construction	Para. 6-4b(2) & 6-4c	Figure 6-22
b. Concrete Floor or roof diaphragm	(1) Inadequate shear capacity	(a) Overlay with new reinforced concrete slab	Para. 6-4b(7)(b)	Figure 6-16
	(2) Inadequate chord capacity	(a) Add new reinforced concrete chord	Para. 6-4b(7)(b)	Figs. 6-16, 6-17
c. Spread footings	(1) Inadequate load capacity	(a) Underpin existing footings	Para. 6-4b(8)(a)	Figure 6-20
		(b) Remove and replace with new footings	Para. 6-4c	Figure 6-22
d. Pile or drilled pier footings	(1) Inadequate load capacity	(a) Add additional piles or piers. Remove and replace existing caps.	Para. 6-4b(8)(b)	Figure 6-21

Table 6-4. Strengthening options for steel moment-resisting frame buildings. (Sheet 1 of 2)

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
a. Steel frames	(1) Inadequate lateral load capacity	(a) Strengthen existing columns	Para. 6-4b(1)(a)	Figure 6-2
		(b) Strengthen existing beams	Para. 6-4b(1)(b)	Figure 6-3
		(c) Modify existing simple beam connections	Para. 6-4b(1)(d)	Figure 6-5
		(d) Add diagonal bracing	Para. 6-4d(3)	Sim. to Fig. 6-4
		(e) Add eccentric bracing	Para. 6-4d(3)	Figure 6-26
		(f) Add new interior or exterior concrete shear walls	Para. 6-4d(2)(a) & (b)	Figure 6-24
		(g) Structural addition(s) to building	Para. 6-4d(6)	
b. Concrete floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with new reinforced concrete slab	Para. 6-4b(7)(b)	Figs. 6-16, 6-18
		(b) Add new horizontal steel bracing	Para. 6-4d(4)	Figure 6-27
	(2) Inadequate shear transfer	(a) Add shear studs	Para. 6-4b(7)(b)	Sim. to Fig. 6-19
	(3) Inadequate diaphragm chord	(a) Modify existing spandrel connections	Para. 6-4b(1)(d)	Figure 6-6

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Table 6-4. Strengthening options for steel moment-resisting frame buildings. (Sheet 2 of 2)

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
c. Steel deck floor or roof diaphragms	(1) Inadequate shear capacity	(a) Additional welding of deck to beams	Para. 6-4b(7)(c)	BDM (Figs. 5-19 & 5-20)
		(b) Add concrete fill and shear studs	Para. 6-4b(7)(c)	Figure 6-19
		(c) Add new horizontal steel bracing	Para. 6-4d(4)	Figure 6-27
d. Timber floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with plywood	Para. 6-4b(7)(a)	Figure 6-14
		(b) Add new horizontal steel bracing	Para. 6-4d(4)	Figure 6-27
	(2) Inadequate shear transfer	(a) Bolt timber decking to steel frame	Para. 6-4b(7)(a)	Figure 6-15
		(b) Bolt timber joists or blocking to steel frame	Para. 6-4b(7)(a)	Figure 6-15
e. Spread footings	(1) Inadequate load capacity	(a) Underpin existing footings	Para. 6-4b(8)(a)	Figure 6-20
		(b) Remove and replace with new footings	Para. 6-4c	Figure 6-22
f. Pile or drilled pier footings	(1) Inadequate load capacity	(a) Add new additional piles or piers. Remove and replace existing caps.	Para. 6-4b(8)(b)	Figure 6-21

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carry-over factors for moment distribution than prismatic members. This may tend to increase the beam moments over the values for the unmodified beams and should be carefully checked to avoid undesirable overstress at critical sections of the beam. The capacity of steel beams in rigid frames may, in some cases, be governed by lateral stability considerations. Although the upper flange may be supported for positive moments by the floor or roof system, the lower flange must be checked for compression stability in regions of negative moments in accordance with section 1.6.1.4 of the AISC Specification. Although properly designed secondary floor beam connections may provide adequate lateral support for those frame beams supporting these secondary beams, beams in frames that are parallel to the secondary beams may need lateral support for the lower flanges in compression due to negative moments. The necessary lateral support may be provided by diagonal braces to the floor system.

(c) Bracing. Strengthening of existing steel bracing, in place, is a viable alternative, provided that the connections, foundations, and other members of the bracing systems are adequate or can also be strengthened to provide the necessary additional capacity. Strengthening of beams and columns in bracing systems can be accomplished as discussed in paragraphs *(a)* and *(b)* above, and strengthening of bracing members that are designed to act only in tension can be accomplished by simply increasing the cross-sectional area of the brace. In strengthening bracing that will act in both tension and compression, it is desirable to strenghten the bracing in a manner that will improve the slenderness ratio, l/r, as well as increase the cross-sectional area. For existing single angle bracing, this may be done by adding an additional angle, back to back, to provide a double angle bracing system. For existing double angle bracing, an additional pair of angles may be added to provide a "starred" section. Typical strengthening details for bracing are shown in figure 6-4.

(d) Connections. Development of a feasible scheme for strengthening the existing connections may be the deciding factor as to whether it is practicable to strengthen existing deficient steel framing. Figure 6-5 indicates how an existing simple beam connection can be modified to resist moment. Spandrel beams in perimeter frames are sometimes required to provide the necessary tension or compression chords for floor or roof diaphragms. If these existing beams are framed to the columns with only simple connections, the flexibility of the connection in tension may result in excessive cracking of the diaphragm. Figure 6-6

indicates how an existing simple beam connection in a spandrel beam can be modified to provide positive chord action for diaphragm. Columns also can be modified to provide increased moment capacity at their base, but the capacity of the base detail needs to be investigated for resistance to the additional moment and horizontal shear resulting from these modifications. Assuming that the foundation is adequate (see paragraphs 6-4*b*(4) or 6-4*c*(4) for modification or replacement of existing footings), the maximum allowable bearing stress under the base plate or the tensile stresses in the anchor bolts may govern the moment capacity at the column base. These stresses are governed by the size of the base plate and the number and configuration of the anchor bolts. While it may be possible to strengthen the column and to stiffen the base plate against local bending, it is usually not practicable to increase the size of the base plate or the number of anchor bolts without removal and replacement of the base plate. The horizontal column shears may be transferred to the column footing by shear lugs between the base plate and the footing, and/or shear in the anchor bolts, and to the ground by passive pressure against the side of the footing. If the column base connection is embedded in a monolithic concrete slab, the slab may be considered for distribution of the shear to the ground by means of any additional existing footings that are connected to the slab.

(2) *Concrete frames.* Strengthening of existing concrete frames is not considered practicable because of the difficulty associated with providing the necessary confinement and shear reinforcement in the beams, columns, and the beam-column panel zones. When deficiencies are identified in these frames, the forces and displacements resisted by these frames can be reduced to acceptable limits by the addition of new structural members (e.g., new frames, shear walls, or bracing) as indicated in paragraph 6-4*d*.

(3) *Reinforced concrete or masonry walls or piers.*

(a) Walls with openings. Existing reinforced concrete or masonry walls with openings may exhibit deficiencies (e.g., excessive shear stresses) in the piers between the openings and/or in the connecting beams between the piers formed by the openings.

1. If the deficiency is in both the piers and the connecting beams, the wall may be strengthened by the addition of reinforced concrete on one or both sides of the existing wall as indicated in figure 6-7. Shallow, highly stressed connecting beams may have to be replaced with properly reinforced concrete as part of the additional wall section. The new concrete may be

Table 6-5. Strengthening opotions for steel braced frame buildings. (Sheet 1 of 2)

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
a. Steel bracing	(1) Inadequate axial load capacity	(a) Increase effective area of braces	Para. 6-4b(1)(c)	Figure 6-4
		(b) Remove and replace with larger bracing	Para. 6-4c	
		(c) Add additional braces	Para. 6-4d(3)	
		(d) Add exterior or interior concrete shear walls	Para. 6-4d(2)(a) and (b)	Figure 6-24
		(e) Structural addition(s) to building	Para. 6-4d(6)	
b. Concrete floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with new rein-forced concrete slab	Para. 6-4b(7)(b)	Figs. 6-16, 6-18
		(b) Add new horizontal steel bracing	Para. 6-4d(4)	Figure 6-27
	(2) Inadequate shear transfer	(a) Add shear studs	Para. 6-4b(7)(b)	Sim. to Figure 6-19
	(3) Inadequate diaphragm chord	(a) Modify existing spandrel connections	Para. 6-4b(1)(d)	Figure 6-6
c. Steel deck floor or roof diaphragms	(1) Inadequate shear capacity	(a) Additional welding of deck to beams	Para. 6-4b(7)(c)	BDM (Figs. 5-19, 5-20)
		(b) Add concrete fill and shear studs	Para. 6-4b(7)(c)	Figure 6-19
		(c) Add new horizontal steel bracing	Para. 6-4d(4)	Figure 6-27

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Table 6-5. Strengthening options for steel braced frame buildings. (Sheet 2 of 2)

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
d. Timber floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with plywood	Para. 6-4b(7)(a)	Figure 6-14
		(b) Add new horizontal steel bracing	Para. 6-4d(4)	Figure 6-27
	(2) Inadequate shear transfer	(a) Bolt timber decking to steel frame	Para. 6-4b(7)(a)	Figure 6-15
		(b) Bolt timber joists or blocking to steel frame	Para. 6-4b(7)(a)	Figure 6-15
e. Spread footings	(1) Inadequate load capacity	(a) Underpin existing footings	Para. 6-4b(8)(a)	Figure 6-20
		(b) Remove and replace with new footings	Para. 6-4c	Figure 6-22
f. Pile or drilled pier footings	(1) Inadequate load capacity	(a) Add additional piles or piers. Remove and replace existing caps.	Para. 6-4b(8)(b)	Figure 6-21

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Table 6-6. Strengthening options for heavy timber frame buildings.

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
a. Posts and beams	(1) Inadequate lateral load capacity	(a) Add diagonal bracing	Para. 6-4b(5)(a)	Figure 6-11
		(b) Add knee bracing	Para. 6-4b(5)(a)	Figure 6-12
b. Timber floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with plywood	Para. 6-4b(7)(a)	Figure 6-14
	(2) Inadequate shear transfer	(a) Provide blocking for nailing to diaphragm and frame members	Para. 6-4b(7)(a)	Figure 6-11
	(3) Inadequate chord capacity	(a) Provide continuous steel members for chord action	Para. 6-4b(7)(a)	Figure 6-14
		(b) Provide continuous timber members for chord action	Para. 6-4b(7)(a)	BDM (Figure 5-33)
c. Spread footings	(1) Inadequate load capacity	(a) Underpin existing footings	Para. 6-4b(8)(a)	Figure 6-20
		(b) Remove and replace with new footings	Para. 6-4c	Figure 6-22
d. Pile or drilled pier	(1) Inadequate load capacity	(a) Add additional piles or piers. Remove and replace existing caps.	Para. 6-4b(8)(b)	Figure 6-21

Table 6-7. Strengthening options for wood stud framed buildings.

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
a. Wood stud walls	(1) Inadequate lateral load capacity	(a) Add plywood sheathing	Para. 6-4b(5)(b)	BDM (Figs. 5-32, 5-34, 6-15, Table 5-6)
		(b) Add let-in bracing	Para. 6-4b(5)(b)	Figure 6-13
	(2) Inadequate tie-down capacity	(a) Add new steel tie-down straps	Para. 6-4b(5)(b)	BDM (Figure 6-16)
b. Timber floor or roof diaphragms	(1) Inadequate shear capacity	(a) Overlay with plywood	Para. 6-4b(7)(a)	Figure 6-14
	(2) Inadequate shear transfer	(a) Provide blocking and nailing, as necessary	Para. 6-4b(7)(a)	BDM (Figs. 5-33, 6-15)
	(3) Inadequate drag struts	(a) Provide new drag struts	Para. 6-4b(7)(a)	BDM (Figure 5-34)
c. Spread footings	(1) Inadequate load capacity	(a) Underpin existing footings	Para. 6-4b(8)(a)	Figure 6-20
		(b) Remove and replace with new footings	Para. 6-4c	Figure 6-22
d. Pile or drilled pier footings	(1) Inadequate load capacity	(a) Add additional piles or piers. Remove and replace existing caps.	Para. 6-4b(8)(a)	Figure 6-21

Table 6–8. Strengthening options for steel stud framed buildings.

Structural Element	Deficiency	Strengthening Technique	Reference	Applicable Figure
a. Steel studs	(1) Inadequate shear capacity	(a) Add new steel straps	Para. 6-4b(6)	BDM (Figure 6-17a)
b. Steel deck roof or floor diaphragms	(1) Inadequate shear capacity	(a) Provide additional welding	Para. 6-4b(7)(c)	BDM (Figs. 5-19 & 5-20)
		(b) Provide concrete fill and shear studs	Para. 6-4b(7)(c)	Figure 6-19
	(2) Inadequate shear transfer	(a) Provide new steel members and welding	Para. 6-4b(7)(c)	BDM (Figure 6-17b)
	(3) Inadequate chord capacity	(a) Provide new steel members and welding	Para. 6-4b(7)(c)	BDM (Figure 6-17b)
c. Spread footings	(1) Inadequate load	(a) Underpin existing footings capacity	Para. 6-4b(8)(a)	Figure 6-20
		(b) Remove and replace with new footings	Para. 6-4c	Figure 6-22
d. Pile or drilled pier footings	(1) Inadequate load capacity	(a) Add additional piles or piers. Remove and replace existing caps.	Para. 6-4b(8)(a)	Figure 6-21